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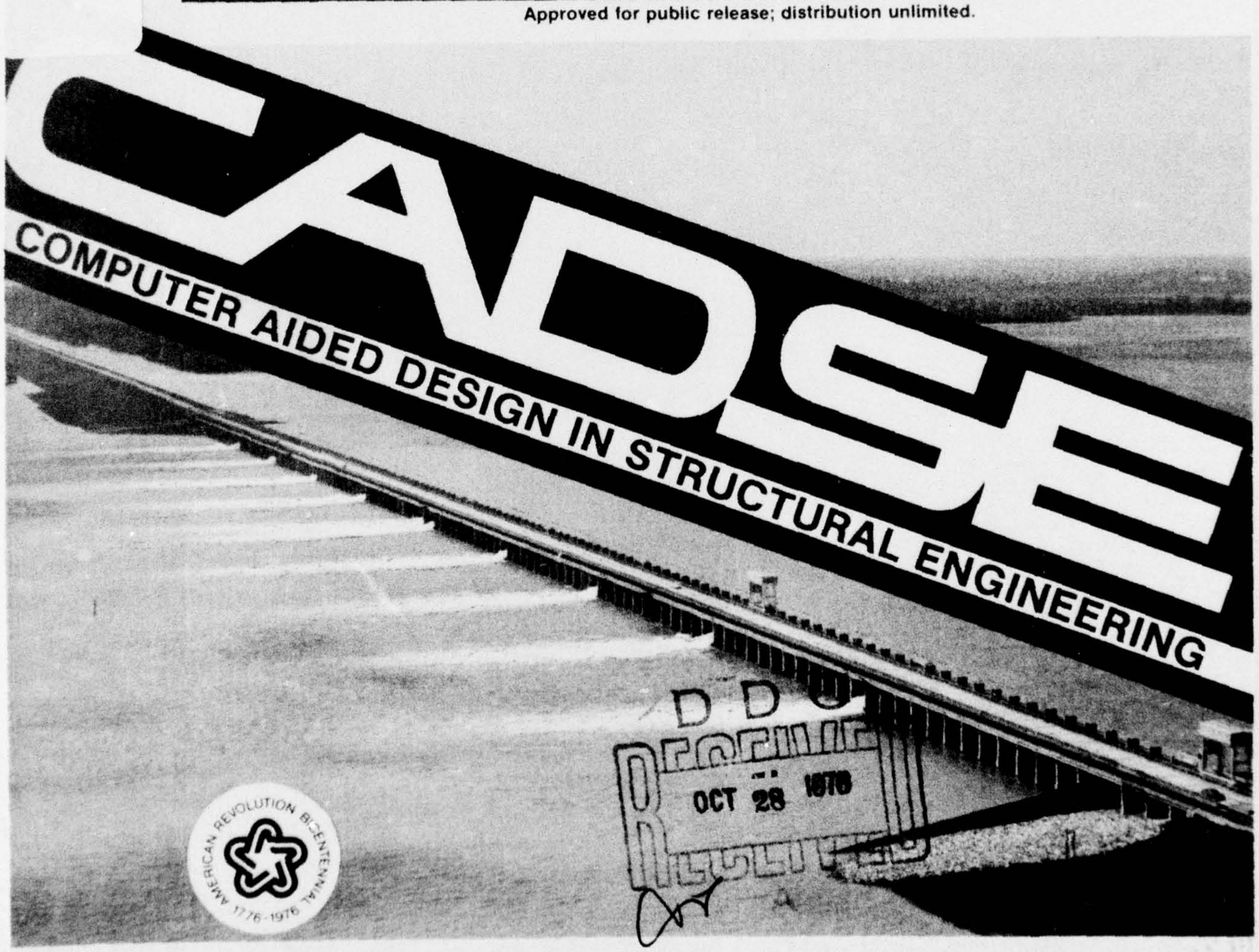
22-26 September 1975

VOLUME IX
SHEET PILE WALLS and T-WALLS

Edited by N. RADHAKRISHNAN



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SHEET PILE WALLS - Cantilever and tieback sheet pile walls can be used for retaining walls and floodwalls. A cantilever sheet pile wall is a continuous diaphragm of interlocking sheet piling supported against lateral loads only by the soil around it. A tieback sheet wall has the same components and is supported by the surrounding soil and exterior supports. The two types of sheet pile walls are compared for suitability in various applications.

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Earth pressure design methods (free earth support and fixed earth support) and the flexible soil support method are compared. Theses design techniques are all suitable for computer applications. A listing of available computer programs is included.

INVERTED T-TYPE FLOODWALLS AND CANTILEVER T-TYPE RETAINING WALLS - Corps of Engineers criteria for the design/analysis of inverted T-type reinforced concrete cantilever floodwalls and retaining walls are summarized. A list of all T-wall computer programs in use throughout the Corps of Engineers is enclosed. CADSE Conference discussions concerning inverted T-type walls are summarized.

PREFACE

In December 1974, the Automatic Data Processing (ADP) Center, U. S. Army Engineer Waterways Experiment Station (WES), submitted a proposal to conduct a Corps-wide Conference on Computer-Aided Design in Structural Engineering (CADSE) to the Office, Chief of Engineers (OCE). OCE approved the proposal and efforts were started in February 1975 to conduct this Conference. The Conference was conducted in New Orleans, Louisiana, 22-26 September 1975 and was attended by 175 engineers from 48 Corps field offices, OCE, Construction Engineering Research Laboratory (CERL), and WES.

This volume contains the papers from Specialty Session E, State-of-the-Corps-Art on Sheet Pile Walls and T-Walls. Mr. Walter D. Judlin III, Chief, Structural Design Section, LMNED-DD, New Orleans District, was session chairman and presented a paper. Mr. Raymond Veselka, Structural Engineer, SWGED-DS, Galveston District, also presented a paper.

The Conference was successful due to the efforts of a multitude of people. The roles they played were different but they were all directed towards making a concept on "instant dissemination" work. The Organizing Committee for the Conference consisted of:

COL G. H. Hilt, WES

Mr. F. R. Brown, WES

Mr. J. B. Cheek, Jr., WES

Dr. N. Radhakrishnan, WES--Conference Coordinator

Mr. W. A. Price, WES

Mr. G. S. Hyde, WES

Mr. D. R. Dressler, LMVD

Mr. W. B. Dodd, LMNDE

Ms. E. Smith, LMNDE

Mr. L. H. Manson, LMNDE

An OCE Coordinating Committee also worked enthusiastically to ensure the success of the Conference. This Committee consisted of:

Mr. C. F. Corns

Mr. R. L. Delyea

Mr. R. F. Malm, OCE Coordinator

Mr. L. G. Gutherie

Mr. D. B. Baldwin

Mr. R. A. McMurrer

The New Orleans District did a remarkable job in playing host to the Conference.

There were 13 Division speakers, 25 moderators, two invited speakers, four technical speakers, and ten session chairmen, who shared the technical load of the Conference. Also, eight computer vendors showed their wares to the participants.

The editors would like to thank all the individuals who served on the committees and the speakers and the moderators for sharing their time and thoughts. Without them the Conference would not have been the success it was. Mr. Dressler, LMVD, and Mr. Price, WES, are specially thanked for their technical guidance and assistance.

This report was edited by Dr. N. Radhakrishnan, Research Civil Engineer, Computer Analysis Branch (CAB) and Special Technical Assistant, ADP Center, under the direct supervision of Mr. J. B. Cheek, Jr., Chief, CAB, and the general supervision of Mr. D. L. Neumann, Chief, ADP center.

The Director of WES during the Conference and the preparation of this report was COL G. H. Hilt, CE. Mr. F. R. Brown was Technical Director.

CONTENTS

SHEET PILE WALLS

by

Walter D. Judlin III*

Introduction

This discussion of sheet pile walls will cover two types, cantilever and tieback. Each sheet pile wall type will be presented as it is used for two applications, retaining walls and floodwalls. As used in this paper, the definition of a cantilever sheet pile wall and a tieback sheet pile wall are as follows: (1) a cantilever sheet pile wall is a continuous diaphragm of interlocking sheet piling (either of steel, concrete, asbestos cement, or whatever material), that derives its total support against lateral loads from the surrounding soil; (2) a tieback sheet pile wall is a continuous diaphragm of the same type of sheet piling whose resistance to lateral load is obtained from the surrounding soil and exterior supports.

General

Both types, cantilever and tieback, are used as retaining walls; the maximum height of cantilever walls is limited to a range of 8 to 15 ft. I favor the lower limit. Excessive deflection caused by creep and plastic movement of the soil results in a long-term tilting and rotation of the wall and may cause distress at the interlocks or monolith joints. This problem is not apparent in tieback walls since restraint is a design assumption. The tiebacks are installed on a sheet pile wall to keep the wall from translating. The wall is restrained at the point of tieback, and the amount of movement of the wall is controlled by the location, number, and strength of the tiebacks. One word of caution on tieback walls: in all cases, the anchor for the tieback should be outside of the zone of failure for the sheet pile wall. That is, the anchor

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* Assistant Chief, Design Branch, New Orleans District.

should be located outside of the active wedge that would develop behind the sheet pile wall should the wall start to fail on a mass stability basis. This anchor should also be far enough away from the active wedge so that the resistance of the anchor would not be reduced by the development of this active wedge. Cantilever walls, when used as retaining walls, have the advantage of being possible to install without structural excavation. A tieback wall can be installed with a minimum of excavation, which is mainly required for the installation of anchors and the tiebacks. In cases where a tieback sheet pile wall will be built and later backfilled, one should be careful to specify that the anchor be backfilled first, before backfilling the sheet pile itself.

Tieback sheet pile walls have limited application as floodwalls. The configuration of sheet pile wall most used as a floodwall is a cantilever wall. The advantages of the I-type floodwall are that it can be built in a very narrow space, requires little or no structural excavation, and is very economical for low wall heights. The Galveston District has built concrete sheet pile I-walls up to 13 ft high. The New Orleans District, using steel sheet piling, has built I-walls in the 8- to 9-ft range and is planning I-walls up to 10-1/2 ft tall. These 10-1/2- and 13-ft I-walls are contradictory to guidance in existing Engineer Regulations; but with sufficient care given to design and consideration of the deflection of the wall, such heights can be achieved safely.

I-wall deflection appears to be the critical design feature, especially when using steel. Up to heights of 10 ft, the New Orleans District (NOD) has never encountered a case where bending or shear is critical; deflection always governs. Where the deflection has been calculated to be excessive, NOD has gone from an I-wall to a pile-supported inverted T-type wall.

There are two basic methods of designing sheet pile walls. First is the earth pressure method; second is the flexible soil support method. Both will be presented so that a direct comparison can be seen.

Earth Pressure Method for Cantilever Wall Design

Cantilever walls rely upon embedment into the soil below the dredge line (in the case of earth retaining walls) or ground line (in the case of floodwalls) to make the wall act as a fixed vertical cantilever beam in resisting applied forces. The applied forces tend to rotate the wall about a pivot point in the embedment depth. (See Figure 1.)

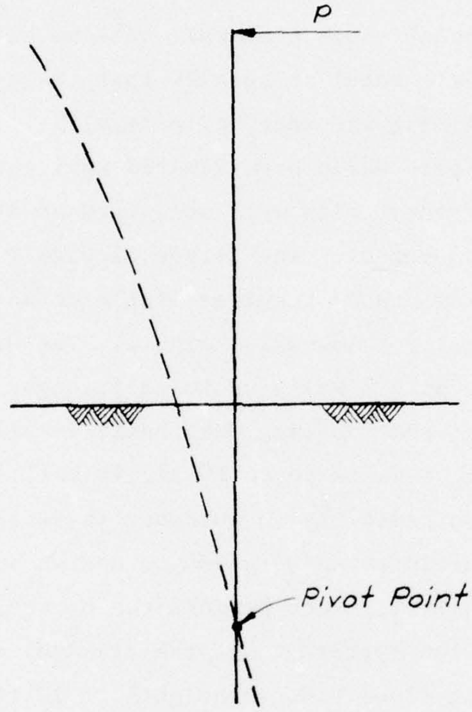

Figure 1. Wall rotation at pivot point due to applied forces

Above the pivot point, passive pressure is developed opposite the direction of the applied forces, and active pressure is developed in the same direction as the applied forces. Below the pivot point, direction of the earth pressures is reversed (since the passive pressure resists movement); passive pressure is developed in the same direction as the applied load and active pressure is developed opposite the direction of the applied load. Thus, the general configuration of the pressure diagram will appear, as shown in Figure 2, to resist the applied load P.

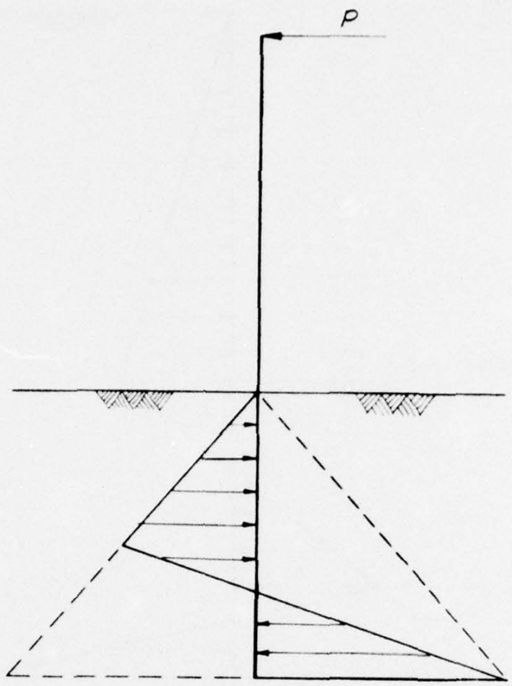

Figure 2. Pressure diagram of wall with applied load

The following discussion of piling penetration determination will be concerned with earth retaining walls. The same general procedures are followed for the design of floodwalls. The major difference between the two is that in the case of earth retaining walls, the applied load P is due to earth pressures; in the case of floodwalls, the applied P is due to hydrostatic pressure.

The method of determining piling penetration follows the general procedure outlined above. The applied load consists of active pressure developed above the dredge line. At the dredge line, passive pressure develops in front of the wall to resist the applied load. Passive pressure behind the wall begins to develop at the pivot point. At the bottom of the wall, the net pressure is equal to the passive pressure behind the wall minus active pressure in front of the wall. The net pressure diagram is thus developed and is shown in Figure 3. The depth of embedment and height Z above bottom can be determined by summing the horizontal

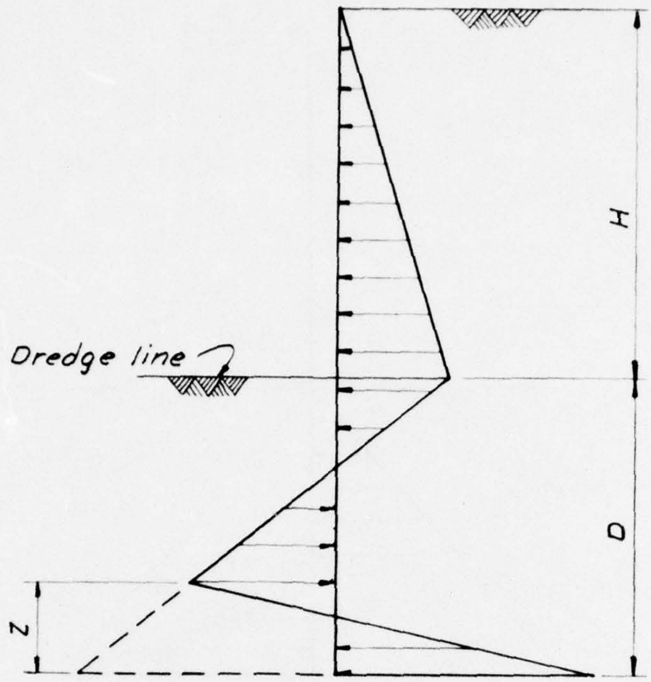

Figure 3. Net pressure diagram

forces and moments about the base and solving two simultaneous equations with two unknowns.

A factor of safety is applied to the penetration by one of the following means:

a. Adding 20 to 40 percent to the calculated depth.

b. Taking 50 to 75 percent of the ultimate passive resistance.

c. Computing and allowable cohesion equal to the actual cohesion divided by a factor of safety, or calculating an allowable angle of friction equal to the arc tan ϕ divided by the factor of safety.

Shears and bending moments are calculated from the net pressure diagram for static equilibrium. Any increase in penetration is not considered in this computation.

Deflections for cantilever walls are calculated as though the wall is fixed at the base and loaded with the net pressure diagram. This is overly conservative.

Earth Pressure Methods for Tieback Wall Design

Tieback walls derive support from soil below the dredge line and the anchor rod. The two earth pressure methods used in design are the free-earth support method and the fixed-earth support method.

Free-earth support method

The free-earth support method is based on the following assumptions:

_a_. Sheet piling is infinitely rigid as compared to the surrounding soil.

_b_. Sheet piling has a hinged support at the anchor rod, i.e., it is free to rotate but not translate. Failure occurs by rotation about the anchor rod.

_c_. No passive pressure develops behind the wall, i.e., no pivot point develops below the dredge line. Active pressure is developed behind the wall and passive pressure develops in front of the wall.

Procedure of design, free-earth support method

Using the free-earth support method, the procedure of design is:

_a_. Determine pressure diagram. (See Figure 4.)

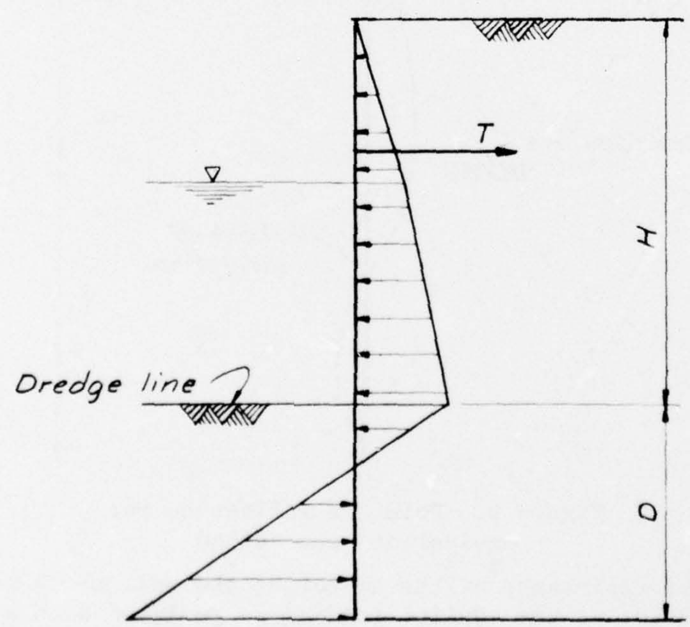

Figure 4. Pressure diagram for tieback wall

b. Solve for depth of penetration by the summation of moments about the anchor rod.

c. Solve for the anchor rod pull by the summation of horizontal forces.

d. Determine the maximum moment at point of zero shear.

Fixed-earth support method

In the fixed-earth support method, the depth of the piling is sufficient to develop a point of fixity at the bottom of the wall. A simplified version of the fixed-earth support method is the equivalent-beam method. The equivalent-beam method is based on the following assumptions:

a. Sheet piling is free to rotate, but not translate, at the anchor rod.

b. Since the bottom of piling is fixed, a point of inflection occurs in the embedment depth. (See Figure 5.)

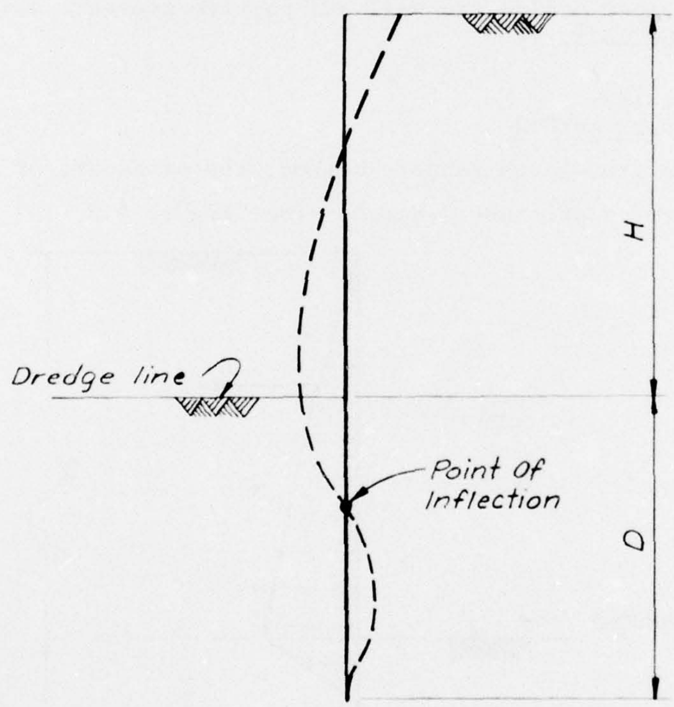

Figure 5. Point of inflection for equivalent-beam method

c. The passive resistance at the bottom of the wall which acts in the same direction as the applied loading is replaced with a concentrated load. (See Figure 6.)

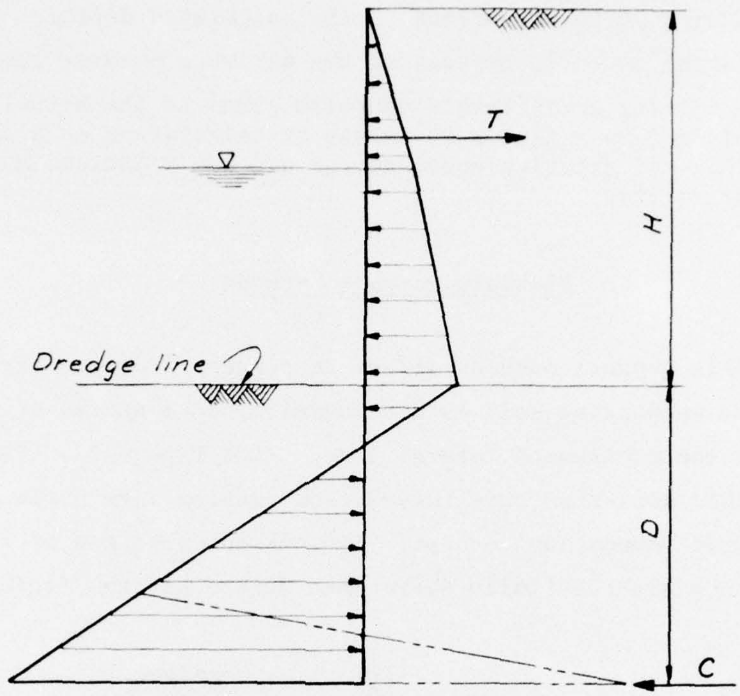

Figure 6. Concentrated load for equivalent-beam method

It should be noted that the equivalent-beam method is valid only for sheet piling walls in granular soil.

Procedure of design,
equivalent-beam method

a. Determine pressure diagram.

b. Locate point of zero pressure on wall.

c. Locate point of inflection. (It may be assumed to be the same as the point of zero pressure.)

d. Determine the horizontal shear at the inflection point by considering the wall above the inflection point a simple beam with reactions at the anchor rod elevation and inflection point.

e. Determine the depth of penetration below the inflection point by considering the wall below the inflection point a simple beam and summing moments about the concentrated load.

f. Determine the maximum moment at the point of zero shear.

A factor of safety is applied to the penetration by one of the following means:

a. Adding 20 to 40 percent to the calculated depth.

b. Taking 50 to 75 percent of the ultimate passive resistance.

c. Computing an allowable cohesion equal to the actual cohesion divided by a factor of safety or calculating an allowable angle of friction equal to the arc tan $\emptyset$ divided by the factor of safety.

Flexible Support Methods

Flexible support methods differ in concept from the earth pressure methods. The supporting soil is considered to be a series of springs which resist the unbalanced lateral load. (See Figure 7.) The methods of solving this soil-structure interaction problem were derived from the beam on elastic foundation concept. Two current versions of flexible support methods are the finite difference method and the finite element method.

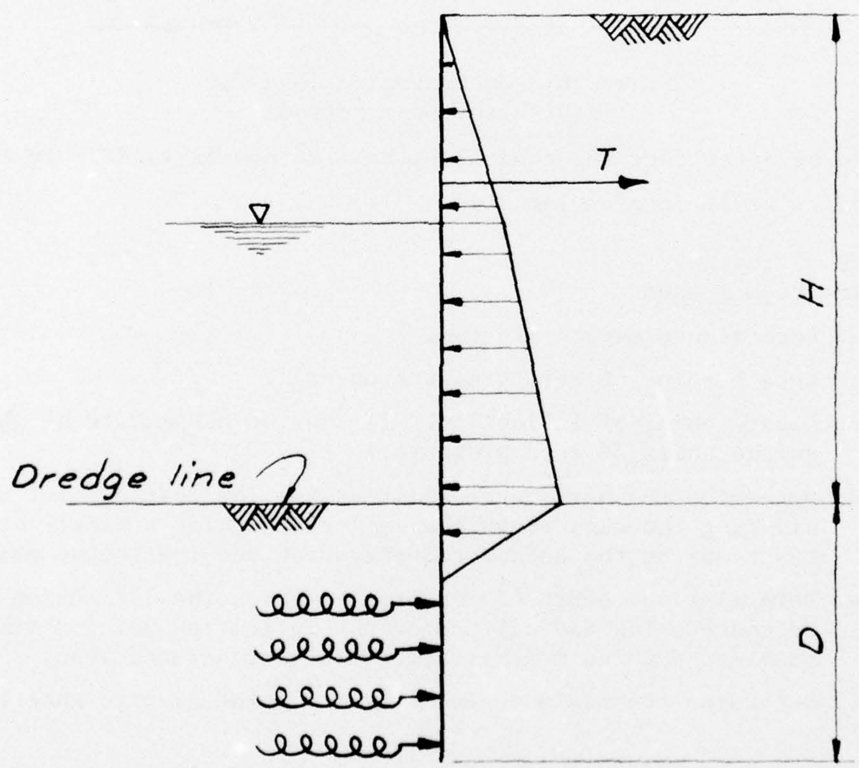

Figure 7. Flexible support method diagram

A discussion of the finite difference method can be found in U. S. Army Engineer Waterways Experiment Station (WES) Miscellaneous Paper K-75-2, "Background Theory and Documentation of Five University of Texas Soil-Structure Interaction Computer Programs." The St. Louis District uses a program by L. Farmer from the University of Missouri at Rolla to solve sheet pile problems using finite difference methods.

A discussion of the finite element method as applied to sheet pile walls is presented in a text by J. Bowles, "Analytical and Computer Methods in Foundation Engineering." Bowles also presents, in this text, a computer program for the analysis of a sheet pile retaining wall by the finite element method. In this program, the soil spring constants are obtained by multiplying the modulus of subgrade reaction at the spring location by the contributing area of the spring. It should be noted that the program solution is not particularly sensitive to significant changes in the modulus of subgrade reaction. Soil pressures resulting from the program solution can be inspected. If it is determined that the soil cannot carry the load, the wall system must be changed. Increasing the embedment does little to reduce the soil pressures, internal moments, or wall deflections. Usually, either a stiffer pile section has to be used, the anchor rod relocated, or the wall height reduced. The Bowles program has been put on time sharing by the New Orleans District and the District is currently modifying the program for floodwalls. The District has not used this program for an actual design, but is still investigating its overall application.

Evaluation of Basic Methods

Of the two basic methods discussed, Engineer Regulations require the use of the pressure method. Both methods have merit and both methods have a place in the design of sheet pile walls. It is the impression of the author that foundations and materials people favor the earth pressure method while structural engineers seem to favor the flexible support method.

As far as the determination of shears, moments, and deflections are

concerned, the flexible support method better predicts the soil-structure interaction and, therefore, will produce truer results. Some authors have compared the flexible support method with field testing and show good correlation between calculated results and measured results.

The earth pressure method ignores the flexibility of the wall. This is a basic reason for overly conservative results. There are factors that can be applied to moments calculated with this method that correct moments based on wall flexibility.

Both methods lend themselves to computer applications. The earth pressure method can evolve back to basic theory; earth pressures can be determined by incremental wedges or planes rather than by derived mathematical expressions. This results in a truer prediction of wall behavior from an overall stability standpoint. Calculation of moments is made by assuming the wall to be a beam with rigid uniformly varying supports and uniformly varying loads.

The flexible support method must, because of the mathematics involved, be performed by computer. The finite difference and finite element methods of calculating the flexible supported wall require, because of the tedium involved, use of computer solutions.

Conclusions

After reviewing both methods of design, I have arrived at the following conclusions:

a. If hand calculation is mandatory, do not consider the flexible support method; use the earth pressure method.

b. Use the earth pressure method to determine penetrations and overall stability.

c. Use the flexible support method for calculations of shears, moments, and deflections.

These conclusions are based on the fact that the penetration of the sheet pile wall is more of a soil problem; earth pressures must be considered to avoid a soil failure. Shear, moment, and deflection are soil-structure interaction problems; therefore, the method that best approximates this interaction should be used. These conclusions also

are consistent with maintaining harmony between the soils engineers and the structural engineers.

If in your particular situation the structural engineer determines the total design, including the soil support, you may want to change these conclusions to suit your own preference. You may choose to use the flexible support method to determine penetrations. If so, please remember to check the resulting soil pressures to insure that you have not exceeded the strength of the soil; also check overall wall stability against a mass failure.

A listing of available computer programs is included as Appendix A.

Appendix A: Computer Program List

ORIGINATOR	PROGRAM NUMBER & TITLE	DESCRIPTION
Vicksburg District	741-F3-A4090 1 ANPI	Anchored sheet pile retaining wall by free earth support method with seepage forces.
Vicksburg District	1 CANPI	Cantilever sheet pile retaining wall with seepage forces.
St. Louis District	713-F3-A350	Solution of beam column with Matlock's recessive solution.
St. Louis District	713-RI-A3-440	*Stability of cantilever* retaining walls determines dimensions according to EM 1110-2-2502.
St. Louis District	713-R1-A3-405	Final design of cantilever retaining walls.
New Orleans District	714-F3-A2-370 Cantilever retaining wall Q & S Case	Determine penetration of cantilever or anchored retaining wall using method of planes.
Detroit District	13-G1-F302 (3003) Newmark Numerical Method for Steel Sheet Piling	Anchored sheet pile retaining wall by four different methods; Free Earth Support, Equivalent Beam, Elastic Line (Fixed Earth), and Equal Moment.

INVERTED T-TYPE FLOODWALLS

AND

CANTILEVER T-TYPE RETAINING WALLS

by

Raymond Veselka*

Introduction

This report summarizes the criteria used by the Corps of Engineers for the engineering design/analysis of inverted T-type reinforced-concrete cantilever floodwalls and retaining walls. A discussion of computer-aided design and the computer programs which are available is included. The report was prepared for presentation at the Corps-wide Conference on Computer-Aided Design in Structural Engineering held in New Orleans on 23-26 September 1975. Appendix A contains a summary of the chief points covered during conference discussions concerning inverted T-type walls.

The function of a floodwall is to prevent inundation and damage to adjacent land and development and to provide for the protection of life during flood periods. The wall is subjected to hydraulic loading on the unprotected side of the wall; it must be designed to safely and economically resist this load. Floodwalls are typically constructed in highly developed industrial, commercial, and residential areas where right-of-way restrictions make levee construction impractical. They have also been used where the foundation is not adequate for stable levee sections. Only inverted T-type floodwalls, similar to the one shown in Figure 1, are discussed in this paper.

A retaining wall is constructed to support a vertical or nearly vertical earth embankment. These walls frequently become a part of the engineering solution to a problem. The only type of retaining wall discussed in this paper is the cantilever T-type retaining wall. This is a reinforced-concrete inverted T-type structure that uses the cantilever

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* Structural Group Leader, Structural Section, Galveston District.

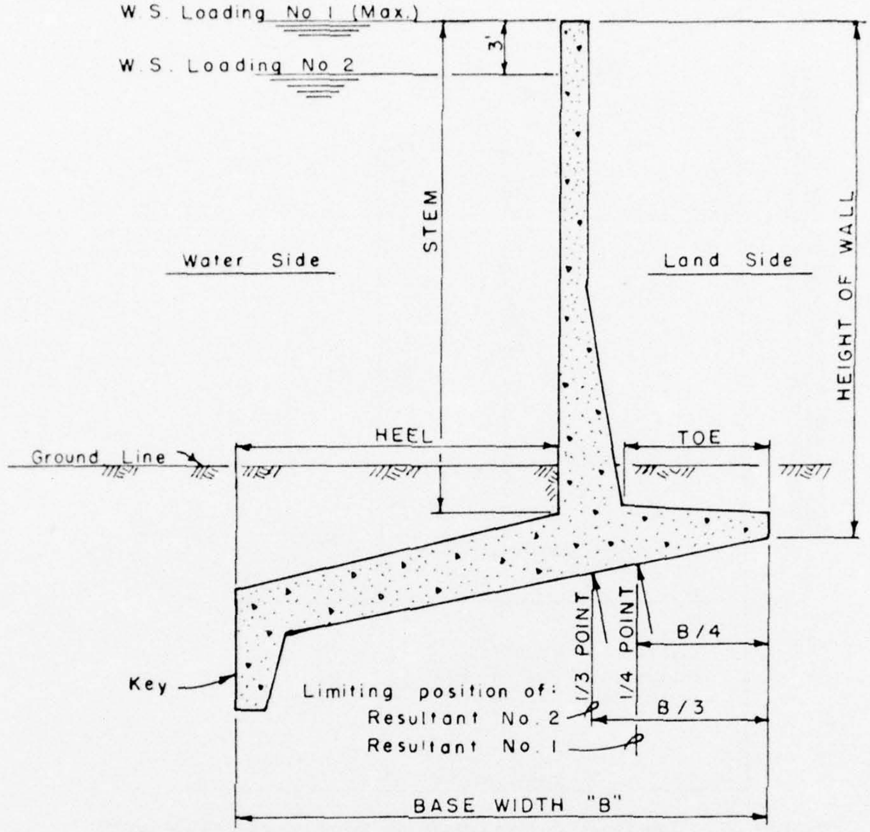

Figure 1. Inverted T-type floodwall

action of its structural components to support the earth loading of the soil mass behind the wall. A typical cantilever retaining wall is shown in Figure 2.

Sources of Design Criteria

To obtain uniformly safe design, the Corps of Engineers has developed engineering manuals (EM's) which provide guidance for planning, design, and construction. EM's and other sources of criteria which apply to the walls discussed herein are as follows:

a. "Working Stresses for Structural Design," EM 1110-1-2101.

b. "Wall Design, Floodwalls," EM 1110-2-2501.

c. "Wall Design Floodwalls," EC 1110-2-156. (Draft copy dated 17 June 1975.)

d. "Retaining Walls," EM 1110-2-2502.

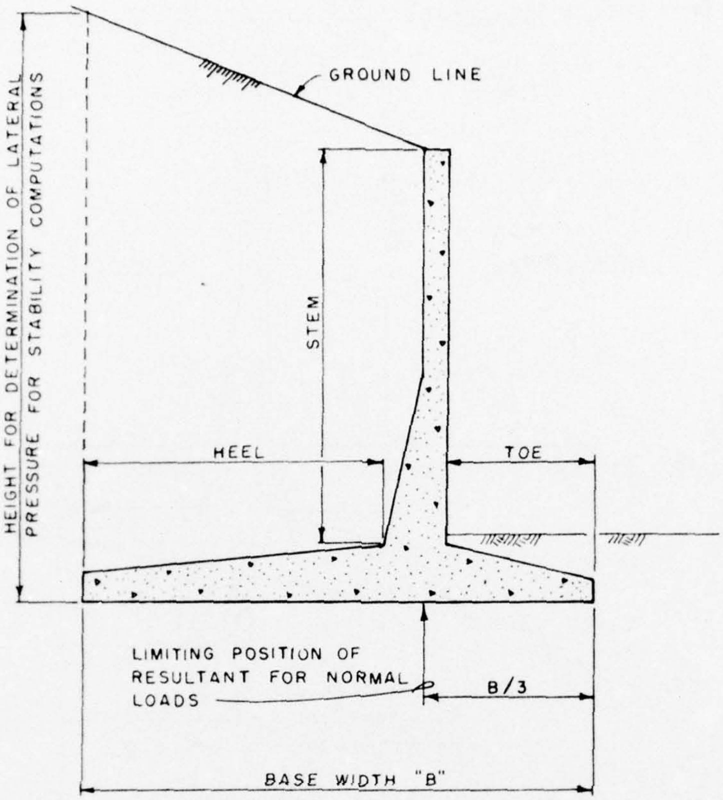

Figure 2. Typical cantilever T-type retaining wall

e. "Design Criteria for Concrete Retaining Walls, August 1971."
United States Department of the Interior
Bureau of Reclamation
Division of Design
Engineering and Research Center
Denver, Colorado

f. "Specifications for Highway Bridges. Section 4: Substructures and Retaining Walls," American Association of State Highway and Transportation Officials, Washington, D. C.

g. "Manual for Railway Engineering. Part 5: Retaining Walls and Abutments," American Railway Engineering Association, Chicago, Illinois.

Discussion of design criteria

An inverted T-type floodwall or cantilever retaining wall may fail in two different ways: (a) its individual members may not be strong enough to resist the applied loads, or (b) the wall as a whole may be unstable and be displaced bodily. The majority of the required wall design effort is expended in satisfying stability requirements. In the

typical design, once stability is assured, all loads are available to design the wall components (stem, toe, heel, and key) as cantilever slabs. Only for high walls with counterforts or buttresses, or for special monoliths, such as floodwall closure structures, does the structural design effort become complete. A thorough discussion of design criteria is beyond the scope of this report. Only a brief summary of some key principles and criteria follows.

Floodwall Design Criteria

Floodwall stability is typically established by considering four possible failure modes: (a) failure by overturning, (b) failure by sliding, (c) failure by exceeding the allowable bearing pressure of the foundation material, and (d) failure by uncontrolled seepage, causing boils and blowouts of the foundation above the toe.

River floodwalls are designed to withstand two flood loading conditions. Loading No. 1 is the maximum possible loading with the water at the top of the stem. Loading No. 2 is the project design load with the water level 3 ft below the top of the stem. In the Galveston District, where floodwalls are designed to resist hurricane tides, loading No. 2 is the load caused by the still-water surge accompanying the standard project hurricane; loading No. 1 is the still-water surge plus the pressure generated by an H_{10} design wave striking the wall.

Percolation of the water through the foundation from the waterside of the wall to the landside creates seepage uplift forces that affect the floodwall stability. Methods commonly used for computing the hydrostatic pressures below the ground include: (a) the flow net method, (b) the line of creep method, and (c) the relaxation method. For overturning structural stability computations, the relatively quick and simple line of creep method is usually sufficiently accurate.

Seepage may be detrimental to the safety of the wall whenever the uplift pressure above the toe of the wall becomes greater than the weight of the submerged earth above the point considered. Under this condition, the foundation material will "boil" and piping will cause loss of foun-

dation material. A design safety factor of at least 2.0 against piping is required. This is often achieved by comparing the wall's creep ratio (defined as the total creep path distance along the buried boundary of the wall divided by the net head between the waterside and landside) to specified values dependent upon the type of foundation material.

A drainage system is required for all river floodwalls to provide a positive seepage outlet to control possible piping. Increasing the concrete heel key length is effective in reducing the quantity of seepage. Whenever the foundation material is highly pervious or is so deep that a concrete cut off key is not practical, one solution of the seepage problem is extension of the key with steel sheet piling.

Leakage at joints between wall monoliths could endanger the foundation of the wall. To prevent excessive joint leakage, a complete waterstop system and limitation of the floodwall's landward movement under repeated loading is specified in the design criteria. By limiting the total wall movement, the differential movement is kept within limits that will not rupture the joint waterstops.

Failure by overturning is checked by summing the overturning and stabilizing movements about the toe of the floodwall. This cannot be done without knowing the point of application of the lateral passive earth pressure on the buried boundary of the structure. Because of the indeterminate nature of the passive pressure distribution, a distribution for design purposes is specified in EM 1110-1-2101. The wall is adequate against overturning when (a) the resultant of the loads induced by loading No. 2 falls within the middle third of the base, and when (b) the resultant induced by loading No. 1 falls within the middle half of the base.

The ultimate bearing capacity is defined as the maximum bearing pressure which an earth or rock foundation material will sustain without exceeding the shearing strength of the material. The allowable bearing pressure suitable for design is the ultimate bearing capacity divided by the desired safety factor. The usual safety factor used for floodwalls is 2.0 for earth foundations. Poor foundation bearing capacity or erosion considerations may require a pile foundation.

Because of the key commonly used with a floodwall, a sliding fail-

ure typically involves failure along a surface within the foundation material itself. The location of the critical failure plane with the lowest safety factor is established by trial analysis of a number of possible planes. Factors of safety are applied to the foundation soil's (a) frictional resisting strength, (b) tangent ϕ , and (c) cohesive strength, until the driving force and resisting forces along the critical plane are equal. The required minimum factors of safety are 1.5 for frictional strength and 1.5 + 2C (C in tons/sq ft) for cohesion.

Wall components are typically designed as cantilever slabs. The reinforced-concrete working-stress theory is used, with the allowable stresses specified for hydraulic structures. The design moments and shears used are the ones caused by loading No. 1, with special considerations involving the point of application of the indeterminate, passive, soil resistance.

Retaining Wall Design Criteria

The behavior of a retaining wall is highly indeterminate. Retaining wall stability is typically established by considering four possible failure modes: (a) failure by overturning, (b) failure by sliding, (c) failure by exceeding the allowable bearing pressure of the foundation material, and (d) failure by sliding along a slip surface separate from the retaining wall.

A major design decision involves the selection of the appropriate values for the lateral pressures which the wall must resist. These pressures are dependent upon the physical characteristics of the soil backfill particles, the soil compaction, the degree of saturation, surcharge loads, soil and structure interaction, and seismic pressures caused by earthquake motion.

Whenever possible, backfill characteristics should be controlled by the use of dense, free draining, nonexpansive soil. This type of backfill material will produce the lowest lateral pressure against the wall. Backfills should be maintained in a drained condition by the use of a positive drainage system.

The relative movement of the wall, with respect to the initial position of the fill, affects the fill pressure to be used in design. Wall movement includes the combined effect of lateral displacement, wall rotation, deflection of members, and foundation yielding. Each design problem must be considered separately. Typically, cantilever retaining walls (backfilled with noncohesive material) that are high and/or founded on an earth foundation will move enough to develop an active state in the backfill. An active soil state produces the lowest limiting value of lateral earth pressure possible. When the wall and foundation are rigid, so that wall movement does not occur, at-rest lateral earth pressures may act on the wall. Jaky's equation ($K_o = 1 - \sin \phi$ may be used to estimate the lateral coefficient for at-rest pressures.

Failure by overturning is checked by summing the overturning and stabilizing moments about the toe of the retaining wall. If the resultant of all normal loading forces on the wall falls within the middle third of the base, adequate safety against overturning exists. With earthquake or temporary construction loads included, the ratio of the stabilizing moments divided by the overturning moments should not be less than 1.15.

Sliding failure can occur along the contact of the base of the wall with the foundation or within the foundation material itself. The factor of safety against sliding is normally defined as the ratio of the resisting forces to the driving forces. This factor of safety should have a minimum value of 1.5 for normal loadings and 1.15 for earthquake or temporary construction loads. Foundation keys are commonly used to increase the stability against sliding.

Computed bearing pressures between the wall and the foundation material are typically compared with allowable bearing pressures to determine if the wall is adequate. The allowable bearing pressure is defined as the ultimate bearing pressure divided by the desired safety factor. The usual safety factors used (for normal wall loads) are 2.0 for earth foundations and 4.0 for rock foundations. Allowable bearing pressures are commonly increased by one-third for earthquake or temporary construction loads. Poor foundation bearing capacity or degradation and

erosion considerations may require the use of a pile foundation.

Computer Aided Design

Both inverted-T floodwall and cantilever retaining wall design techniques start with the selection of tentative dimensions; these are analyzed for stability and structural strength requirements to determine if they satisfy the design criteria. Dimensions are then revised, if necessary, and a new analysis is made. Several cycles of analysis and revision are commonly required to obtain a satisfactory solution.

Design time and expense can be greatly reduced by the use of a computer to perform the analyses computations. The most economical design can be easily obtained by using appropriate program routines to revise the wall base dimensions and structural member sizes until the most suitable solution is achieved.

Results of the Corps-wide survey indicate that approximately 50-60 inverted-T floodwalls and a like number of cantilever retaining walls are presently being designed every year. Computers appear to be used to some degree as an aid in the majority of these designs.

The majority of programs used throughout the Corps to aid in the design of inverted-T floodwalls and cantilever retaining walls do not seem to be very sophisticated; that is, the program performs analysis computations identical to the longhand computational method. These programs simply relieve the engineer from doing tedious hand calculations and provide time for the consideration of more design cases. The programs closely follow the criteria contained in appropriate engineering manuals. Many programs are not general purpose, i.e., they are limited to a specific wall configuration, foundation type, etc. Programs also differ because they contain local design criteria interpretation and design preference. Program documentation quality varies widely. Documentation is often inadequate for the first-time user of the program to determine how to use the program or whether the program is appropriate for the design problem to be solved.

From an overall view, it appears obvious that too many similar

computer programs, which perform the same basic design, have been developed. Because it is difficult to obtain information needed to locate, evaluate, and use programs developed in another design office, it is likely that more unnecessary program development will be done in the future. It is therefore recommended that a competent task force be established to evaluate the existing programs and determine if one or more can be adopted as a Corps-wide standard. The final program or programs should then be added as an appendix to the appropriate engineering manual.

Appendix A: Computer Programs

NOTE: Copies of available program abstracts, writeups and listings may be obtained directly from the program originator.

ORIGINATOR	CANTILEVER RETAINING WALL	INVERTED-T FLOODWALL
Alaska District	713-J3-G1-090, Retaining Wall Stability Analysis	
Albuquerque District		
Baltimore District		
Buffalo District		
Charleston District		
Chicago District		
Detroit District		713-G1-F3-040, T-Wall Design 741-G1-F3-050
Fort Worth District		
Galveston District		713-G1-M3-060, Inverted T Floodwall Stability Design (Primarily for Hurricane Design with wave loading)
Huntington District		
Jacksonville District		
Kansas City District	713-J6-F1-160, Retaining Walls 713-R3-C1-070, Retaining Walls 713-R1-C1-020, Retaining Wall Design Phase V	Unpublished Local Program

Little Rock District		
Los Angeles District	713-X6-L1-002, Design of Channel T-Wall Unpublished Local Program	
Louisville District		741-K2-H2-010, Floodwall Structural Design & Sliding Stability
Memphis District	Unpublished Local Program	FL WALL (Undocumented overturning and sliding analysis).
Mobile District	713-J2-K5-160, Retaining Wall Stability Analysis (Rankin with sloping fill) 713-J2-K5-190, Retaining Wall Stability Analysis (Coulomb Theory, trial wedge)	
Nashville District		
New Orleans District	741-F5-A2-370, Cantilever Retaining Wall Q + S Cases 741-F5-A2-510, Cantilever Retaining Wall Stability Q + S Cases 741-G1-A2-180, Retaining Wall Stability	713-F3-A2-110, Forces on Inverted T-Wall 713-F3-A2-160, Forces on Linear Variation Depth T-Wall
New York District		
Norfolk District		
Omaha District		
Philadelphia District	713-F7-E5-080, Retaining Wall Stability Irregular Base	
Pittsburgh District		
Portland District		

ORIGINATOR	CANTILEVER RETAINING WALL	INVERTED-T FLOODWALL
Rock Island District		713-G1-F442A, Stability of T-Type Floodwall
Sacramento District		
San Francisco District	713-G2-L3-003, Cantilever Retaining Wall	
Savannah District	713-G1-K6-010, Retaining Wall Stability Analysis Revision 713-G1-K6-130, Cantilever Retaining Wall	
Seattle District		
St. Louis District	713-R1-A3-440, Cantilever Retaining Wall Design Stability 713-G1-A3-450, Cantilever Retaining Wall Stability Analysis & Final Design 713-G1-A3-220, Cantilever Retaining Wall Analysis 713-G1-A3-010, T-Type Retaining Wall 713-G1-A3-570, Cantilever Retaining Wall Offset Toe and Heel Slabs 713-R1A3690, Lateral Press on Retaining Wall from Boussinesq for Point Surcharge	713-G1-A313, T-Type Wall Sliding Stability 713-R1-A3080, T-Type Floodwall - Overturning Stability, Load, Shear & Moment Diagrams
St. Paul District	713-G1-F5-040, T-Type Retaining Wall 713-R3-A3-010, T-Type Retaining Wall	713-G1-F5-060, T-Type Floodwall 713-I1-F5-090, Floodwall Stability 713-I1-F5-030, T-Type Floodwall
Tulsa District	713-G1-M5-300, Wall Stability Analysis and Plot	
Vicksburg District	713-G1-A4-090, Cantilever Retaining Wall	
Wilmington District		
Walla Walla District	713-K5-G4-300, Cantilever Retaining Wall	

Missouri River District	Unpublished Local Program	Unpublished Local Program
New England Division		Unpublished Local Program
Pacific Ocean Division	Unpublished Local Program	
Waterways Experiment Station	741-G9-R0-BC0, Cantilever Retaining Wall Stability	
Bureau of Reclamation	Canal Structure Sec, Ultimate Strength Design for Cantilever Walls	
General Electric Civil Engineering Package	G.E.-200 Series - Retaining Wall Design Program No. CD225 P2.012 Publication No. CP8-1068	
Textbook Foundation Analysis and Design - McGraw-Hill Book Co.	Appendix D - Program 6 Design of a Retaining Wall	

Appendix B: Test Problem Solution to Floodwall Stability Using "EFFRAM" Computer Program

by

Philip A. Mattson

INDEX

GENERAL DESCRIPTION

For the purpose of this study, the simplified floodwall shown in Sketch 1 was reduced to finite elements of 5 ft. lengths in both the stem and the base. The geometry of the nodal points between the elements, the member properties, the subgrade modulus, and the standard Load Condition 1 constitutes the input to the EFFRAM computer program.

The following three trial solutions reflect varying conditions of restraints and earth pressures approximating possible modes of soil-structure interaction. The objective of this preliminary investigation was to test the flexibility of the EFFRAM program, particularly the use of lateral and vertical "springs" simulating soul pressures.

Sketch 2 shows the idealized structure and the manner in which the hydrostatic and gravity loads were applied. The computer solution provided the soil pressures generated by these loads, as well as the element shears, moments, displacements, etc.

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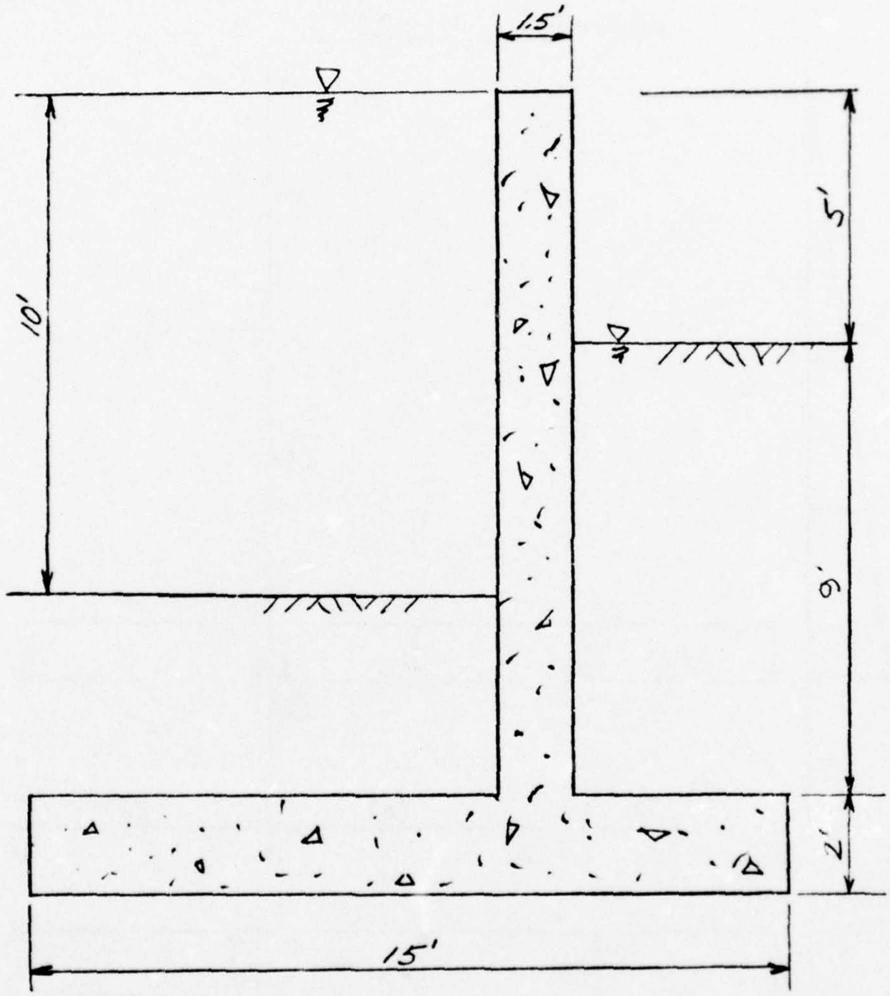
PAGE 2

SUBJECT TEST PROBLEM - FLOODWALL STABILITY BY EFFRAM

COMPUTATION

COMPUTED BY DM CHECKED BY DATE 9/13/75



Section of Test wall
Sketch 1

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PAGE 3

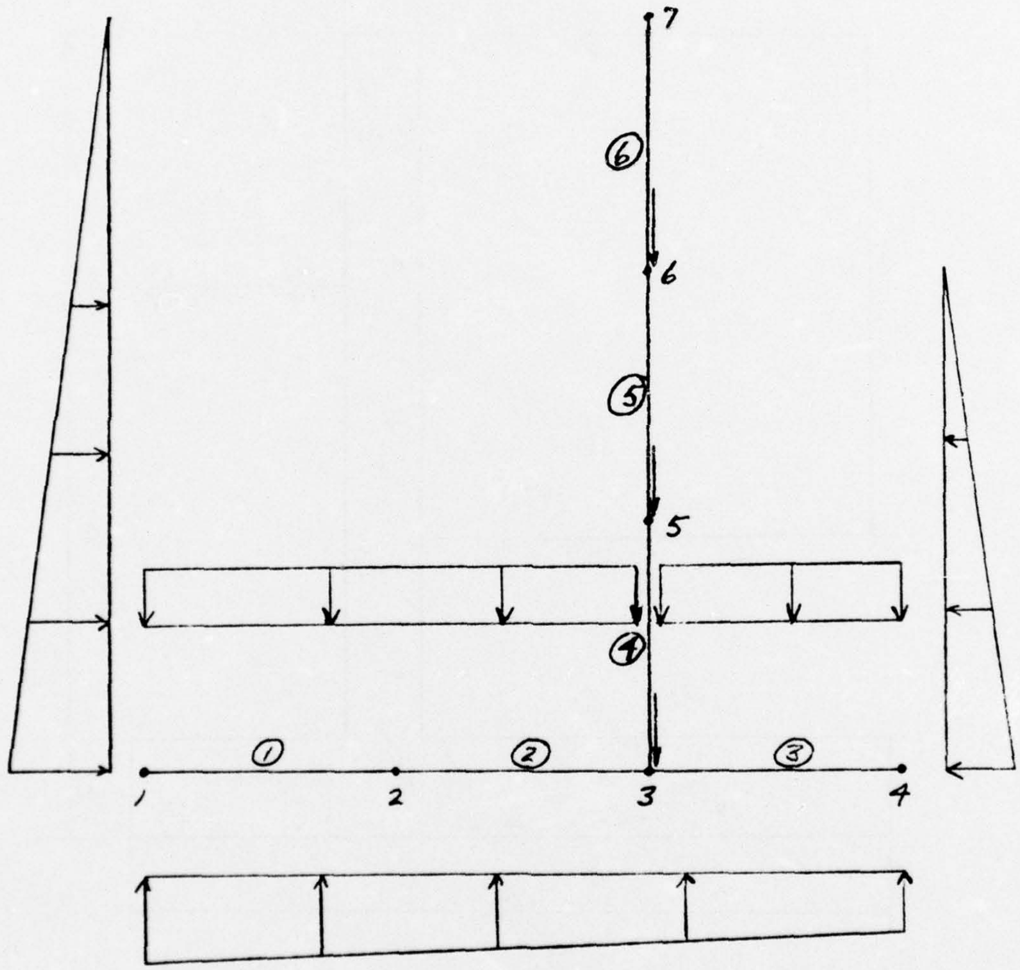
SUBJECT TEST PROBLEM - FLOODWALL STABILITY BY EFFRAM

COMPUTATION

COMPUTED BY PM CHECKED BY DATE 9/19/75

nodal points: •2

elements: ③



Computer Model
Sketch 2

THREE TRIAL SOLUTIONS

In each of the following cases, the external loads acting upon the structure are identical; only the support conditions have been altered.

CASE I. Vertical springs simulating earth pressure reaction have been placed beneath the base, combined with a horizontal restraint on the structure to counter the imbalance in lateral forces.

CASE II. In addition to the vertical springs of Case I, horizontal springs have been applied to the landside of the stem in place of the restraint. (The modulus of subgrade reaction in the lateral direction was assumed to vary linearly from zero at ground elevation to maximum value at base depth.)

CASE III. A horizontal force was applied to the base simulating a hypothetical frictional resistance to sliding. The "earth springs" were unchanged from Case II.

```
L1-
     1        TEST PROBLEM - STABILITY ANALYSIS FLOODWALL
     2        CASE 1 - VERT EARTH SPPINGS UNDER BASE
     3        & X RESTRAINT AT NODE 1
     4        7,6,1
     5        1,100,5*0
     6        2,0,5,4*0
     7        3,0,10,4*0
     8        4,0,15,4*0
     9        5,0,10,5,3*0
    10        6,0 10,10,3*0
    11        7,0,10,15,3*0
    12        1,1,2,1,430000,115,5,3*0
    13        2,2,3,1,430000,115,1000,3*0
    14        3,3,4,1,430000,115,5,3*0
    15        4,3,5,1,430000,0,1000,3*0
    16        5,5,6,1,430000,0,5,3*0
    17        6,5,7,1,430000,0,5,3*0
    18        LOAD CONDITION 1
    19        3,0
    19.1      3,0,-1.13,0
    19.2      5,0,-1.13,0
    19.3      6,0,-1.13,0
    20.1      103,2,-.62,0,1
    20.2      103,4,.187,0,1
    20.3      405,4,-.75,0,2
    20.4      406,4,.938,0,2
    22        5*0
    23         END
 Q
SRU'S:1.3
ITEM NO (1 TO 9) ?5
INPUT FILE NAME ?SUPPRE
SRU'S:1.2
ITEM NO (1 TO 9) ?6
SRU'S:7.3
SRU'S:1.5
ITEM NO (1 TO 9) ?2
EDIT FILE NAME ?EFOUT
L130-*
 ** TIME 14:47:21
 ** SECTION      1
 ** LOADING      1


 ELEMENTS ON AN ELASTIC FOUNDATION

 ELEMENT                SPRING  PRESSURE
                        I              J


      1    0.                  .420542E+00
      2     .420542E+00        .106102E+01
      3     .106102E+01        .169303E+01


 NUMBER OF ITERATIONS =      4


 ELEMENT ZONE      LIMITS OF INTEGRATION (MAX=11)


      1 EVEN       0.     ,     1.710,      5.000,
      2  ODD       0.     ,     5.000,
      3  ODD       0.     ,     5.000,
```

** DATE 09/13/75
** TIME 14:47:21
** SECTION 1
** LOADING 1

NODAL POINT DISPLACEMENTS

NODE	X-DISPL	Y-DISPL	ROT-DISPL
1	0.	.18323E-02	-.11006E-02
2	.34235E-04	-.36254E-02	-.11042E-02
3	.63470E-04	-.91467E-02	-.11043E-02
4	.63470E-04	-.14538E-01	-.10962E-02
5	.55905E-02	-.91702E-02	-.11045E-02
6	.11143E-01	-.91820E-02	-.11133E-02
7	.16715E-01	-.91820E-02	-.11145E-02

END ACTIONS WITH ELEMENT LOADS

ELEMENT	AXIAL I AXIAL J	SHEAR I SHEAR J	MOMENT I MOMENT J
1	-3.287	-.000	.000
	3.287	1.629	-4.914
2	-3.287	-1.627	4.932
	3.287	.564	-11.602
3	0.	-3.955	-11.085
	0.	.000	-.000
4	2.250	3.286	22.682
	-2.250	-2.190	-8.860
5	1.130	2.189	8.860
	-1.130	-.782	-1.303
6	0.	.782	1.303
	0.	-.000	-.000

1

** EFFRAM ** ANALYSIS OF PLANE FRAMES ON ELASTIC FOUNDATIONS

NED FORM 223
27 Sept 49

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SUBJECT TEST PROBLEM - FLOODWALL STABILITY BY EFFRAM

COMPUTATION

COMPUTED BY PM CHECKED BY DATE 9/19/75

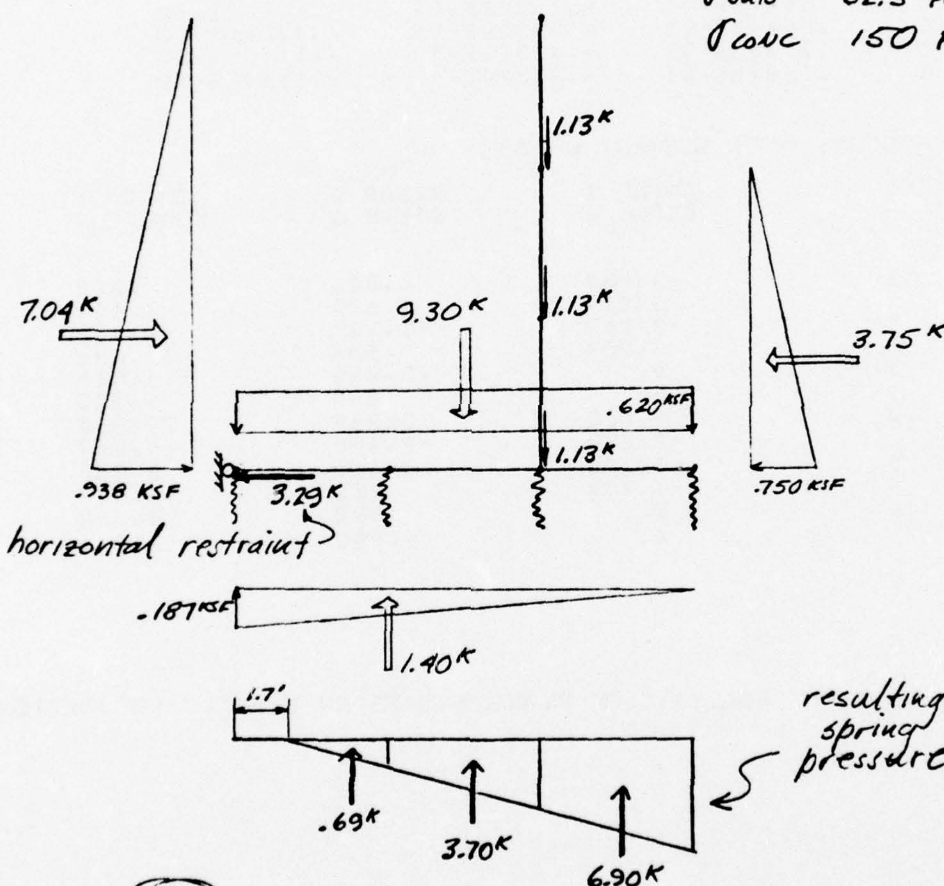
CASE I

γ_{SAT} 125 PCF

γ_{WATER} 62.5 PCF

γ_{SUB} 62.5 PCF

γ_{CONC} 150 PCF



$\Sigma F_H = 0$ (+→) $7.04^K - 3.75^K - 3.29^K = 0$ ✓

$\Sigma F_V = 0$ (↓+) $9.30 + 3(1.13) - 1.40 - .69 - 3.70 - 6.90 = 0$ ✓

```
~L1-
     1        TEST PROBLEM - STABILITY ANALYSIS FLOODWALL
     2        CASE 2 - VERT & HORIZ EARTH PRESSURE
     3        SPRINGS
     4        7,6,1
     5        1,0,5*0
     6        2,0,5,4*0
     7        3,0,10,4*0
     8        4,0,15,4*0
     9        5,0,10,5,3*0
    10        6,0 10,10,3*0
    11        7,0,10,15,3*0
    12        1,1,2,1,430000,116,5,3*0
    13        2,2,3,1,430000,116,1000,3*0
    14        3,3,4,1,430000,116,5,3*0
    15        4,3,5,1,430000,5,1000,3*0
    16        5,5,6,1,430000,2,5,3*0
    17        6,6,7,1,430000,0,5,3*0
    18        LOAD CONDITION 1
    19        3,0
    19.1      3,0,-1.13,0
    19.2      5,0,-1.13,0
    19.3      5,0,-1.13,0
    20.1      103,2,-.62,0,1
    20.2      103,4,.137,0,1
    20.3      405,4,-.75,0,2
    20.4      405,4,.938,0,2
    22        5*0
    23         END
~Q
SRU'S:1.5
ITEM NO (1 TO 9) ?5
INPUT FILE NAME ?SUPPRE
SRU'S:1.2
ITEM NO (1 TO 9) ?6
SRU'S:4.9
SRU'S:1.6
ITEM NO (1 TO 9) ?2
EDIT FILE NAME ?EFOUT
~L130-*
 ** TIME 14:31:42
 ** SECTION      1
 ** LOADING      1


ELEMENTS ON AN ELASTIC FOUNDATION

ELEMENT              SPRING   PRESSURE
                     I               J


       1      .155930E+00     .553342E+00
       2      .553342E+00     .952420E+00
       3      .952420E+00     .134859E+01
       4      .434544E+00     .505162E+00
       5      .163337E+00     .175309E+00

NUMBER OF ITERATIONS =      0

ELEMENT ZONE      LIMITS OF INTEGRATION (MAX=11)

       1   ODD       0.    ,      5.000,
       2   ODD       0.    ,      5.000,
       3   ODD       0.    ,      5.000,
       4   ODD       0.    ,      5.000,
       5   ODD       0.    ,      5.000,
```

** DATE 09/18/75
** TIME 14:31:42
** SECTION 1
** LOADING 1

NODAL POINT DISPLACEMENTS

NODE	X-DISPL	Y-DISPL	ROT-DISPL
1	.80757E-01	-.13447E-02	-.68551E-03
2	.80757E-01	-.47745E-02	-.68719E-03
3	.80757E-01	-.82105E-02	-.68722E-03
4	.80757E-01	-.11525E-01	-.68162E-03
5	.84194E-01	-.82341E-02	-.68731E-03
6	.87555E-01	-.82458E-02	-.59517E-03
7	.91133E-01	-.82458E-02	-.59585E-03

END ACTIONS WITH ELEMENT LOADS

ELEMENT	AXIAL I AXIAL J	SHEAR I SHEAR J	MOMENT I MOMENT J
1	0.	-.000	-.000
	0.	.547	-2.066
2	0.	-.538	2.066
	0.	-.586	-2.651
3	.000	-2.810	-7.719
	-.000	.000	.000
4	2.260	.038	10.354
	-2.260	-1.310	-6.763
5	1.130	1.330	5.696
	-1.130	-.781	-1.302
6	.000	.782	1.303
	-.000	-.000	.000

1

** EFFRAM ** ANALYSIS OF PLANE FRAMES ON ELASTIC FOUNDATIONS

NED FORM 223
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CORPS OF ENGINEERS, U. S. ARMY

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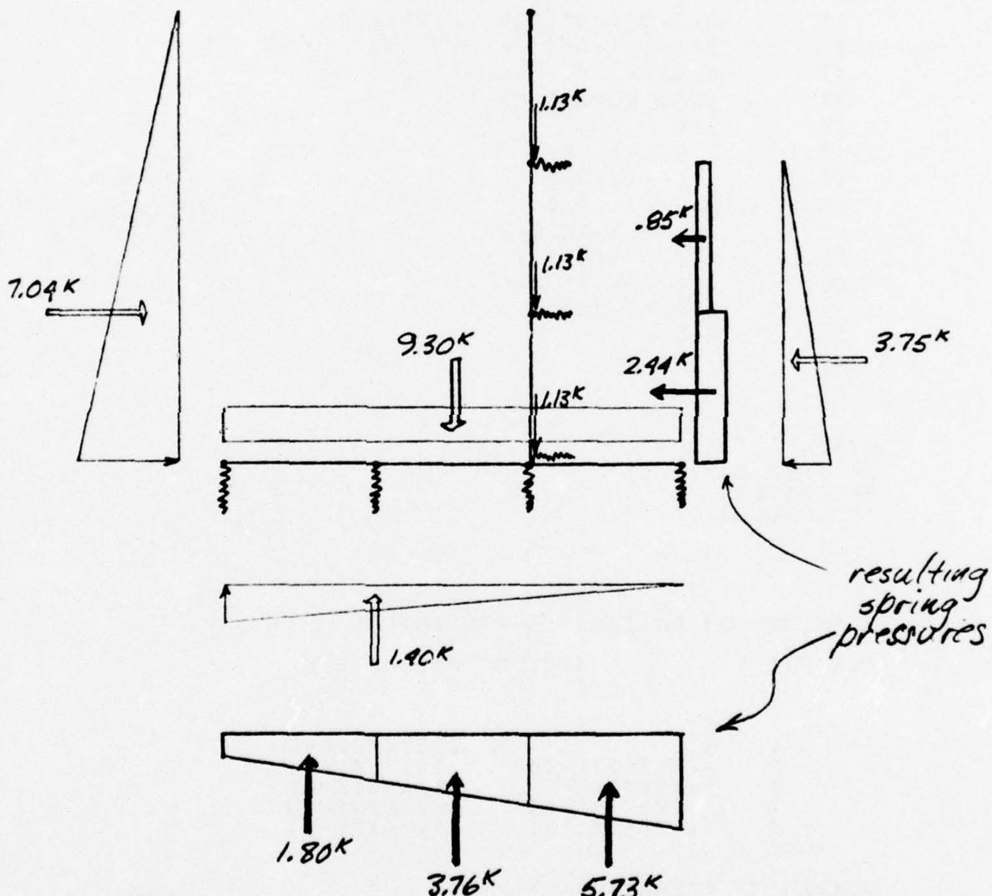
SUBJECT TEST PROBLEM - FLOODWALL STABILITY - BY EFFRAM

COMPUTATION

COMPUTED BY PM CHECKED BY DATE 9/17/75

CASE II



$\Sigma F_H = 0$ (+→) $7.04^K - 3.75 - 2.44 - .85 = 0$ ✓

$\Sigma F_V = 0$ (↓+) $9.30^K + 3(1.13) - 1.40 - 1.80 - 3.76 - 5.73 = 0$ ✓

```
  1      TEST PROBLEM - STABILITY ANALYSIS FLOODWALL
  2      CASE 3 - VERT & HORIZ EARTH SPRINGS
  3      WITH FRICTION FORCE ON BASE
  4      7,6,1
  5      1,0,5*0
  6      2,0,5,4*0
  7      3,0,10,4*0
  8      4,0,15,4*0
  9      5,0,10,5,3*0
 10      6,0 10,10,3*0
 11      7,0,10,15,3*0
 12      1,1,2,1,430000,116,5,3*0
 13      2,2,3,1,430000,116,1000,3*0
 14      3,3,4,1,430000,116,5,3*0
 15      4,3,5,1,430000,5,1000,3*0
 16      5,5,6,1,430000,2,5,3*0
 17      6,6,7,1,430000,0,5,3*0
 18      LOAD CONDITION 1
 19      3,0
 19.1    3,-2,-1.13,0
 19.2    5,0,-1.13,0
 19.3    6,0,-1.13,0
 20.1    103,2,-.52,0,1
 20.2    103,4,.137,0,1
 20.3    405,4,-.75,0,2
 20.4    406,4,.933,0,2
 22      5*0
 23       END


L130-*
** TIME 14:16:25
** SECTION       1
** LOADING       1


ELEMENTS ON AN ELASTIC FOUNDATION

ELEMENT                 SPRING   PRESSURE
                          I            J

        1    0.                    .437379E+00
        2     .437379E+00          .101764E+01
        3     .101764E+01          .154441E+01
        4     .175263E+00          .202673E+00
        5     .675575E-01          .757433E-01

NUMBER OF ITERATIONS =     2

ELEMENT ZONE      LIMITS OF INTEGRATION (MAX=11)

        1 EVEN      0.    ,      .334,      5.000,
        2  ODD      0.    ,     5.000,
        3  ODD      0.    ,     5.000,
        4  ODD      0.    ,     5.000,
        5  ODD      0.    ,     5.000,
```

** DATE 09/18/75
** TIME 14:16:25
** SECTION 1
** LOADING 1

NODAL POINT DISPLACEMENTS

NODE	X-DISPL	Y-DISPL	ROT-DISPL
1	.29211E-01	.34969E-03	-.91030E-03
2	.29211E-01	-.42059E-02	-.91336E-03
3	.29211E-01	-.37723E-02	-.91342E-03
4	.29211E-01	-.13314E-01	-.90641E-03
5	.33779E-01	-.37953E-02	-.91356E-03
6	.33374E-01	-.33031E-02	-.92227E-03
7	.42938E-01	-.33031E-02	-.92295E-03

END ACTIONS WITH ELEMENT LOADS

ELEMENT	AXIAL I AXIAL J	SHEAR I SHEAR J	MOMENT I MOMENT J
1	0.	.000	.000
	0.	1.195	-3.941
2	0.	-1.192	3.945
	0.	.072	-8.033
3	.000	-3.453	-9.624
	-.000	.000	-.000
4	2.250	1.999	17.625
	-2.250	-1.315	-8.024
5	1.130	1.828	7.938
	-1.130	-.781	-1.303
6	.000	.782	1.303
	-.000	.000	-.000

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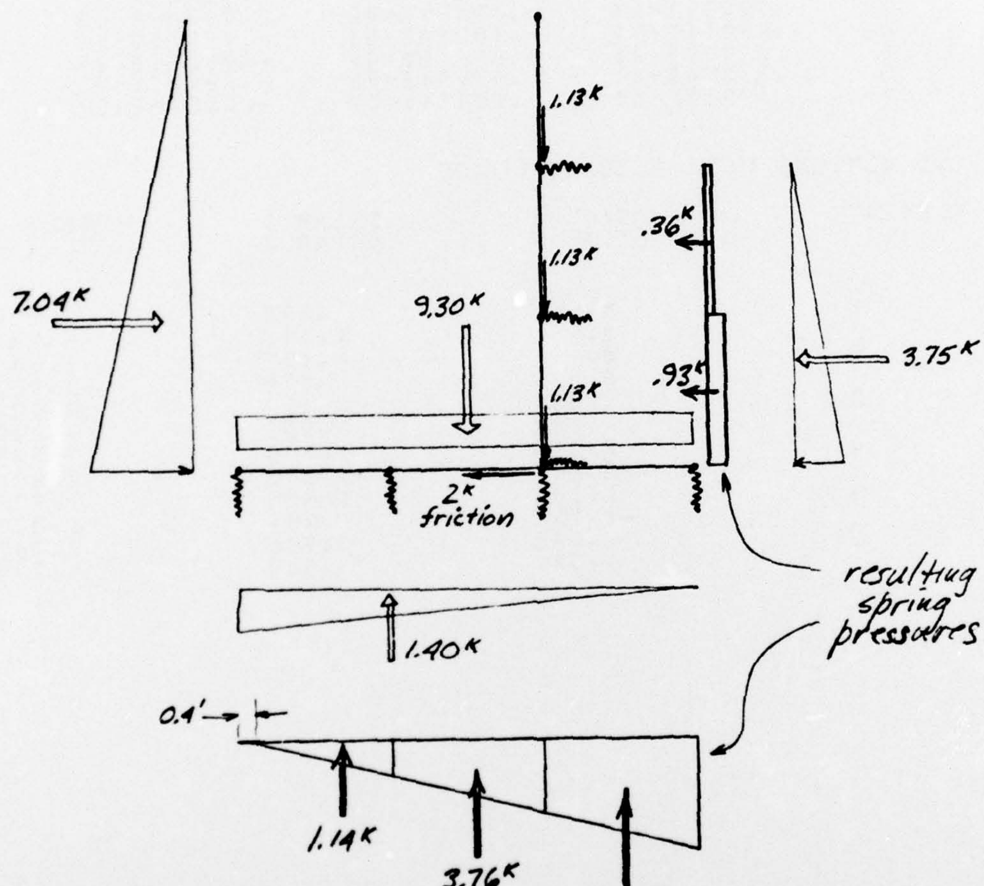
SUBJECT TEST PROBLEM - FLOODWALL STABILITY BY EFFRAM

COMPUTATION

COMPUTED BY PM CHECKED BY DATE 9/11/75

CASE III



$\Sigma F_H = 0$ (+→) $\quad 7.04 - 3.75 - 2 - .36 - .93 = 0$ ✓

$\Sigma F_V = 0$ (↓+) $\quad 9.30 + 3(1.13) - 1.40 - 1.14 - 3.76 - 6.40 \doteq 0$ ✓

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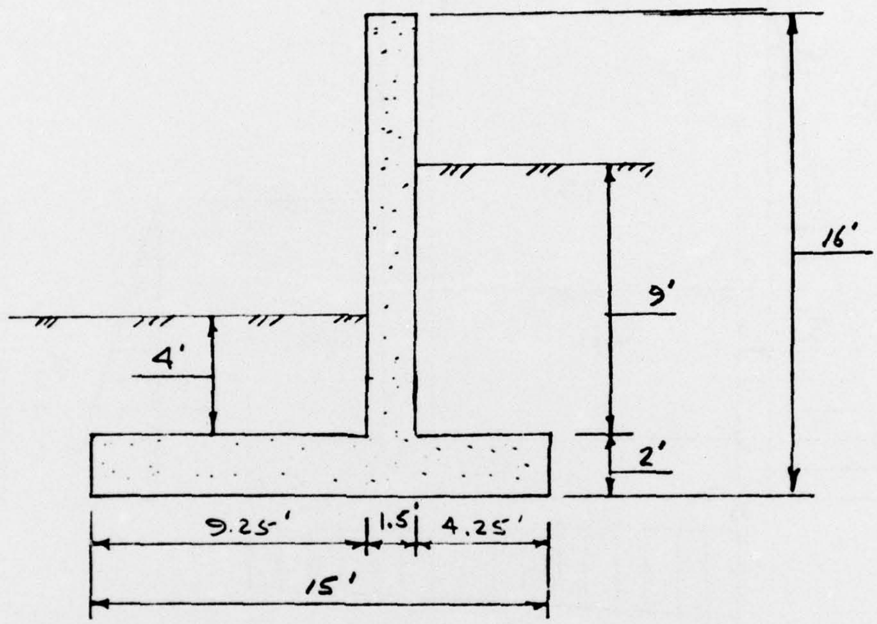
SUBJECT TEST PROBLEM
COMPUTATION STABILITY
COMPUTED BY JRF CHECKED BY DATE 9/19/75



DESIGN CONSTANTS:

γ_{SAT}	=	125.0	pcf	$K_A = 0.33$
γ_{SUB}	=	62.5	"	$K_P = 3.00$
γ_{WAT}	=	62.5	"	$\mu = 0.30$
γ_{CONC}	=	150.0	"	

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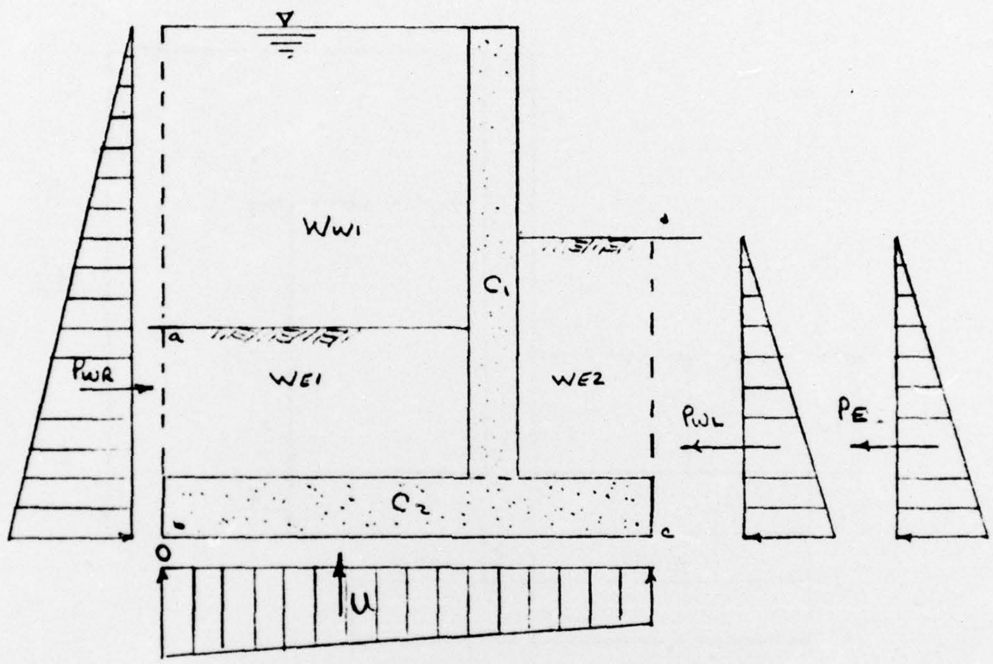
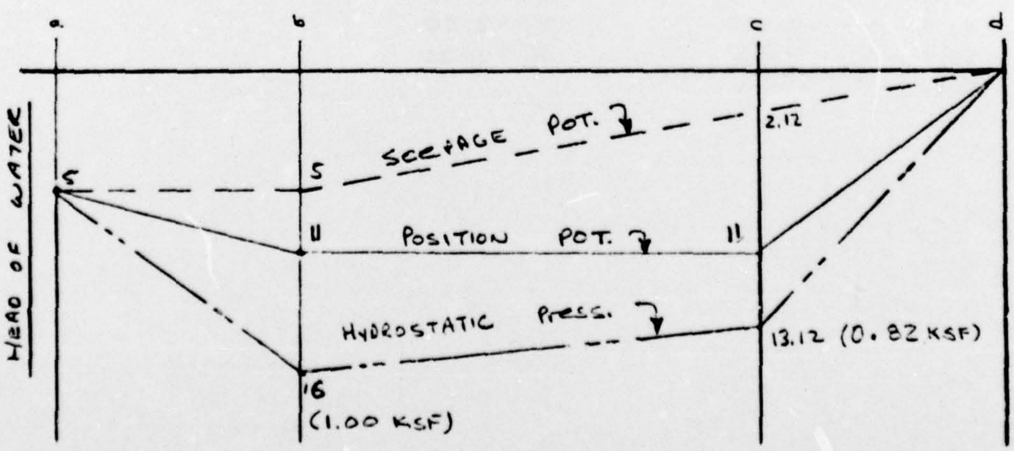
LOADING CONDITION 1 - WATER LEVEL AT TOP OF STEM



CREEP DIAGRAM



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SECT.	FORCE COMPUTATION	VERTICAL (K) ↓	↑	HORIZONTAL (K) →	←	ARM (FT)	MOMENT (K-FT) ↻	↺
C1	$.15 \times 1.5 \times 14$	3.15				10.0	31.50	
C2	$.15 \times 2.0 \times 15$	4.50				7.5	33.75	
Ww1	$.0625 \times 9.25 \times 10$	5.78				4.62	26.70	
WE1	$.125 \times 9.25 \times 4$	4.62				4.62	21.34	
WE2	$.125 \times 4.25 \times 9$	4.78				12.88	61.57	
U	$\frac{15}{2}(1.00 + 0.82)$		13.65			7.25		98.96
P_{WR}	$.0625 \times 16^2 \times \frac{1}{2} \times$			8.00		5.33	42.64	
P_{WL}	$\frac{1}{2} \times .0625 \times 13.12^2$				5.38	3.67		19.74
P_E	$\frac{1}{2} \times .0625 \times .33 \times 11^2$				1.25	3.67		4.59
Σ	————	22.83	13.65	8.00	6.63	—	217.50	123.29
		9.18 ↓		1.37 →			94.21 ↻	

$$\frac{\Sigma H}{\Sigma V} = \frac{1.37}{9.18} = 0.15$$ ∴ ACTIVE Pressure

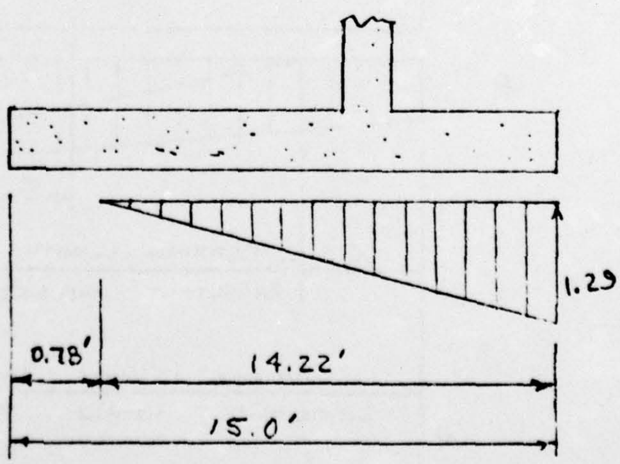
$$\frac{\Sigma M}{\Sigma V} = \frac{94.21}{9.18} = 10.26 \text{ ft}$$ ∴ RESULTANT IS WITHIN THE MID-HALF (The Resultant Ratio is 0.32)

BEARING Pressures

$$P_E = \frac{2\Sigma V}{3m}$$

$$P_E = \frac{2 \times 9.18}{3 \times 4.74}$$

$$P_E = 1.29 \text{ KSF}$$



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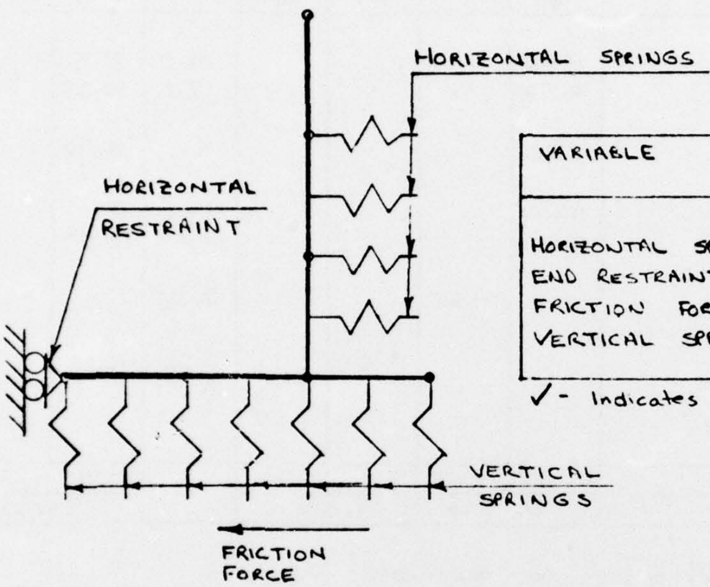
SUBJECT: TEST PROBLEM
COMPUTATION: SUMMARY OF RESULTS
COMPUTED BY: VRF   CHECKED BY: DATE: 9/19/75

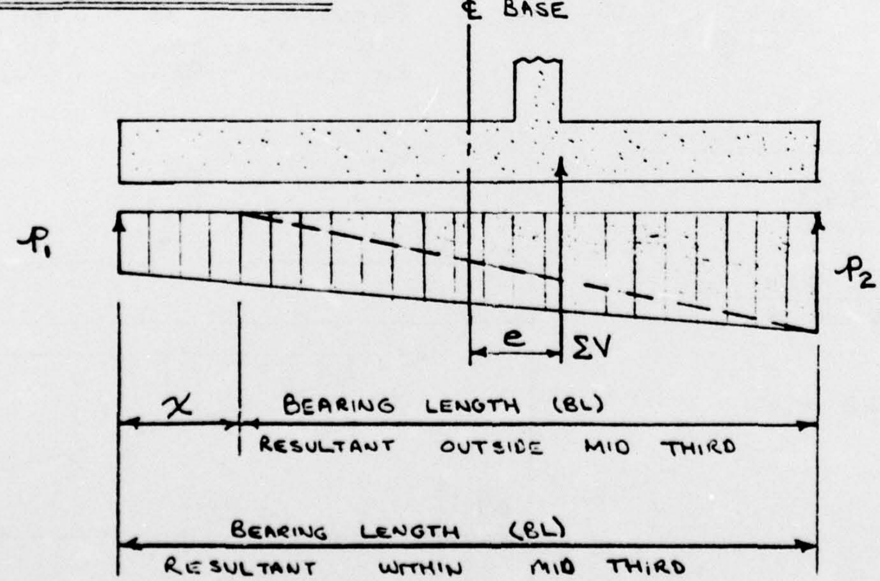
VARIABLES IN EFRM ANALYSIS



VARIABLE	EFRM CASE I	EFRM CASE II	EFRM CASE III
HORIZONTAL SPRINGS		✓	✓
END RESTRAINT	✓		
FRICTION FORCE			✓
VERTICAL SPRINGS	✓	✓	✓

✓ - Indicates variable is included.

BEARING PRESSURES



SUBJECT Test Problem

COMPUTATION Summary of Results

COMPUTED BY JRF CHECKED BY DATE 9/19/75

SUMMARY OF RESULTS

ITEM	EFRM I	EFRM II	EFRM III	HAND COMPUTATION
p_1 (KSF)	0.00	0.16	0.00	0.00
p_2 (KSF)	1.70	1.34	1.54	1.29
BL (FT)	13.32	15.00	14.66	14.22
x (FT)	1.68	0.00	0.34	0.78
e (FT)	3.07	1.97	2.61	2.76

APPENDIX A:

SUMMARY OF CONFERENCE DISCUSSIONS

Four sessions on sheet pile walls and T-walls were held on 24 and 25 September 1975. These sessions were attended by a total of 85-90 participants from various Corps of Engineers offices. Approximately 45 minutes of each 1-1/2 hour session was devoted to the combined subjects of inverted T-floodwalls and retaining walls. The sessions were intentionally oriented toward computer-aided design of these structures. After a brief introduction of the subject the session was opened to discussion by the participants. Included below is a summary of the principal items discussed.

There were several discussions that involved the availability, adequacy, and future need for computer programs for inverted T-floodwalls and retaining walls. The principal discussion themes are as follows:

There needs to be some organized system established to disseminate information about computer programs.

Program documentation, in general, needs to be greatly improved. No standard documentation rules are enforced. Documentation is often not done because it requires time and funds, and program development is now done with bootleg funds from projects.

Standard Corps of Engineers programs for both inverted-T floodwalls and retaining walls should be adopted. This would best be accomplished by a funded task force. The task force would evaluate the existing programs, and have the authority to adopt standard criteria where there are differences of opinion. The final program should handle as many of the usual design conditions as possible and should be interactive and/or a design program, not an analysis program. The final adopted programs should be added as an appendix to the appropriate EM. Design by use of a standard accepted program would simplify the review process for walls.

Because of the long lead time required to get a standard program, a list of suggested programs to be used at the discretion of the designer should be made available as soon as possible.

There was some sentiment that the programs for inverted-T floodwalls and retaining walls are not that complex and that standard programs would inhibit the freedom that now exists to develop programs for special conditions.

There appeared to be general agreement that program development should be done by special application personnel in the Design Branch, not by ADP personnel.

Design criteria were not discussed extensively. The following is a summary of the criteria and technical design questions discussed.

The subject of finite element analysis procedures for T-walls was introduced. A paper presenting results of some work done in the New England Division on this subject is included in Appendix B of this report. The author does not believe that the design methods used by the Corps of Engineers for these structures should be changed at this time. However, it appears that the finite element analysis procedure could be used as a research tool to check the validity of many of the traditional design assumptions currently in use. If this is to be done, a more realistic representation of the structure, backfill, and foundation soils structure interaction must be used than the one employed in the New England Division's work (see Appendix B).

The subject of ultimate strength versus working stress design for reinforced concrete was discussed. One opinion is that ultimate strength design was developed for building design and the method is not appropriate for large concrete sections, as used in civil works structures. Recent engineering graduates and most A-E firms are oriented to use ultimate strength design. Several new design criteria were added when American Concrete Institute (ACI) code 318-71 was adopted. This author believes that a new EM 1110-1-2101 is needed to clarify Corps of Engineers criteria.

The use of wall friction in determining active lateral earth pressure for the design of retaining walls was discussed. The general consensus was that wall friction is not now used in the design of retaining walls. The effect of including wall friction does not greatly reduce the lateral pressure. Often uncertainty about the proper soil properties to use in wall design overshadows this consideration.

There was one discussion about the appropriate uplift values to use for hurricane floodwall design. The Galveston District generally considers two conditions: (a) the uplift caused by the design surge

still-water level, and (b) the condition of no uplift, based on the assumption that the impervious soil and short time duration of the surge does not develop uplift.

APPENDIX B: BIOGRAPHICAL SKETCHES

Walter D. Judlin III, is at present the Assistant Chief of the Design Branch, New Orleans District, New Orleans, Louisiana. He earned a Bachelor of Science in Civil Engineering from Tulane University in 1960. He is a registered professional engineer in the State of Louisiana and an affiliate member of the American Society of Civil Engineers. He began his employment with the New Orleans District in March 1960 as a Structural Engineer in the Engineering Division and has since held the positions of Chief, General Engineering Section (Nov 1973-March 1975), and Chief, Structural Design Section (March 1975-May 1976) before occupying his present position.

Raymond Veselka is the Structural Group Leader, Galveston District, Galveston, Texas. He earned a Bachelor of Science in General Engineering at Texas A&M, another in Civil Engineering at the University of Houston, and a Mechanical Engineering degree in Civil Engineering at Texas A&M.

In accordance with ER 70-2-3, paragraph 6c(1)(b), dated 15 February 1973, a facsimile catalog card in Library of Congress format is reproduced below.

Corps-Wide Conference on Computer-Aided Design in Structural Engineering, New Orleans, La., 1975.

[Proceedings ... held in New Orleans, Louisiana, 22-26 September 1975] Vicksburg, Miss., Automatic Data Processing Center, U. S. Army Engineer Waterways Experiment Station, 1976-

12 v. illus. 27 cm.

Contents.-v.1. Management report.-v.2. List of computer programs for CADSE.-v.3. Invited speeches and technical presentations.-v.4. Division presentations.-v.5. State-of-the-Corps-Art (SOCA) reports on gravity monoliths, U-frame locks, and channels.-v.6. SOCA reports on gates, stoplogs, and trashracks.-v.7. SOCA reports on single- and multiple-cell conduits and tunnels.-v.8. SOCA reports on pile foundations and sheet pile cells.-v.9. SOCA reports on sheet pile walls and T-walls.-v.10. SOCA reports on stiffness methods, frames, and military construction.-v.11. SOCA reports on earthquake and dynamic analyses.-v.12. Interactive graphics, SEARCH and CORPS systems.

(Continued on next card)

Corps-Wide Conference on Computer-Aided Design in Structural Engineering, New Orleans, La., 1975.

[Proceedings ...] 1976-

(Card 2)

1. Computer-aided design -- Congresses. 2. Design -- Congresses. 3. Structural engineering -- Congresses. I. U. S. Army. Corps of Engineers. II. U. S. Waterways Experiment Station, Vicksburg, Miss.

TA641.C67 1975